IS 7894 (1975): Code of practice for stability analysis of earth dams [WRD 9: Dams and Spillways]

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Indian Standard

CODE OF PRACTICE FOR STABILITY ANALYSIS OF EARTH DAMS

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Indian Standard

CODE OF PRACTICE FOR STABILITY ANALYSIS OF EARTH DAMS

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Indian Standard

CODE OF PRACTICE FOR STABILITY ANALYSIS OF EARTH DAMS

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 10 December 1975, after the draft finalized by the Dam Sections (Non-overflow) Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Earth embankments are widely used for roads, railways, river training works, canal embankments, dams, etc. The economy and safety of these works can be accomplished by adopting proper methods of design, construction and maintenance. The failure of these structures is likely to result in loss of life and damages to property. It may also result in damage and/or washout of the structure fully or partially.

0.3 The most important cause of failure is sliding. It may occur slowly or suddenly and with or without forewarning. Such a failure causes a portion of the earth fill to slide downwards and outwards with respect to the remaining part generally along a well-defined slide surface. At the time of the failure the average shearing stress exceeds the average shearing resistance along the sliding surface. It is, therefore, necessary that the designer takes special care to eliminate the possibility of such a failure.

0.4 In the design of earth dam both safety and economy call for thorough soil studies of the foundation and of the materials of construction, combined with stability computations. The methods of stability analysis currently in use have been developed largely as a result of studies of actual slides on old dams. The stability computations serve as a basis either for the redesign of slope of an existing structure or for deciding the slope of a new structure in accordance with the specified safety requirement. Because of wide variations in the properties of subsoil formation and the heterogeneity of soils available for construction of earth dam, the design of an earth dam constitutes a problem that calls for individual treatment. Additional studies are required in complex situations such as earth dams founded on marshy soils, marine clays and materials susceptible to liquifaction.

0.5 Effective stress method of analysis is recommended in this code.

0.6 For the purpose of this standard it is presumed that the requirements given below have been determined before carrying out the stability analysis

of the proposed dam section:

a) Zoning pattern of the dam for the most economical utilization of the available material in the appropriate zones preceded by a thorough soil exploration for strength, permeability, etc.

b) Design shear strength parameters and densities for the various zones in the section.

c) The geological features and strength parameters of the foundation material that have a bearing on the stability of the embankment.

d) Tentative section of the dam required for examining stability on the basis of judgement and past experience for more or less similar conditions.

e) Top level of dam, maximum water level (MWL), full reservoir level (FRL), low water level (LWL) that is level beyond which lake will not be depleted and tail water level (TWL) finalized on the basis of project requirements.

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1. SCOPE

1.1 This standard covers the procedure for analysing the stability of earth dam slopes for different conditions of loading to which the dam is expected to be subjected during its life time. The analysis covers types of shear test results to be used, conditions requiring analysis, computation of pore pressures, methods of analysis and considerations for earthquake forces.

1.2 This standard does not cover special cases such as transition between earth dam and masonry and concrete dam.

1.3 This standard does not cover stability analysis of rockfill dams.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

2.1 Actuating or Driving Force — The force which tends to cause movement or sliding of the soil mass in the dam.

2.2 Critical Surface — The possible failure surface which gives the lowest value of factor of safety.

2.3 Effective Stress Method — A method of analysis in which the pore pressures that exist on the potential failure surface within the dam and the foundation are estimated and allowed for separately from normal stress. The shear strength of soils are also determined in terms of effective stress.

2.4 Embankment Dam — It includes earth dams and earth core rock-fill dams.

2.5 Factor of Safety — For circular arc analysis, it is the ratio of stabilizing force to actuating force. For sliding wedge analysis, it is the ratio of design shear strength to developed shear strength.

2.6 Failure Surface — The plane or surface along which a part of the dam section tends to move or slide.

2.7 Phreatic Line — The upper flow line, or free surface of seepage, along which the pressure in the soil water is equal to the atmospheric pressure.

2.8 Stabilizing or Resisting Force — The force which opposes movement or sliding of soil mass in the dam.

2.9 Steady Seepage — This corresponds to a condition in which steady seepage flow has been established through the dam section for any reservoir level.

2.10 Sudden Drawdown — That rate of lowering of reservoir water level which does not allow full dissipation of pore pressure simultaneously with the lowering of reservoir level.

2.11 Total Stress Method — A method of analysis in which total stresses, without allowance for pore water pressures, are taken into account. The shear strength of the soils adopted in analysis are also determined in terms of total stresses without pore pressure measurements. In this procedure it is assumed that the pore pressures which will develop in the sample during the laboratory test will be equal to those which would develop in the dam at the time of failure. In other words the shear strength is assumed to include the influence of the pore pressure during loading as well as shearing.

2.12 Trial Sliding Mass — The portion of the dam and foundation (if any) lying above the assumed failure surface.

3. PRINCIPLES

3.1 Whenever difference of levels exists within a continuous soil mass, gravitational forces tend to cause the movement from the higher elevation to the lower one. Another force which tends to cause this movement is the force due to seepage. Earthquakes also generate forces which tend to cause movement. All these forces cause shearing stress throughout the soil and a mass movement occurs unless the resisting forces on every possible surface, plane or curved, are more than the actuating forces.

3.2 The shear strength at failure on any surface within an earth dam is directly related to the normal stress on that surface and has the

relationship expressed by Coulomb's equation:

$$S = C' + N' \tan \phi'$$

$$S = C' + (N - U) \tan \phi'$$

where

S = shear strength of the failure surface,

C' = cohesion intercept in terms of effective stress,

N = total normal stress acting on the failure surface,

U = pore water pressure acting on the failure surface,

$N' = N - U$ = effective normal stress, and

ϕ' = angle of shearing resistance in terms of effective stress.

4. DESIGN SHEAR STRENGTH

4.1 All the shear test results for earth dam and foundation material adopted in the stability analysis should be those obtained from triaxial shear testing as far as possible. The embankment shear strength shall be obtained by performing the tests on the proposed embankment material compacted to dry density aimed at during the construction of dam. The foundation strength shall be estimated after testing the undisturbed soil samples from the foundation or from the results of *in-situ* tests. The shear testing shall be done from zero to maximum normal stress expected in the dam.

4.1.1 Adequate number of representative samples of each type of soil to be used in the earth dam shall be tested for shear strength. The design shear parameters for the materials comprising the dam shall be fixed at a value such that 75 percent of the test results are above the design value. The extreme or freak values shall, however, be rejected.

4.1.2 The foundation materials shall be classified in accordance with IS : 1498-1970*. Adequate number of samples of each type of foundation material shall be tested for shear strength. The design shear parameters for each type shall be fixed so that at least 75 percent of the test results are above the design value. The extreme or freak values shall, however, be rejected.

4.1.3 The density values for the material shall be fixed at 75 percent reliable value and the moisture content corresponding to that value shall be adopted for calculations.

4.1.4 For the purpose of this standard, the various types of shear tests performed under different conditions of loading and drainage shall be designated by letters Q, R and S as follows:

Q test = unconsolidated undrained test,

R test = consolidated undrained test, and

S test = consolidated drained test.

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*Classification and identification of soils for general engineering purposes (*first revision*).

4.2 Q Test — During this test, initial water content is kept constant. Thus the change in the water content is not permitted either prior to or during shear. For saturated soils as the drainage of water from the voids does not take place, the applied normal load induces a balanced pore water pressure and hence the shear strength is independent of the normal load and envelope of the shear strength is a straight line parallel to the abscissa. For partially saturated soil samples, however, there is a decrease in the volume due to the compression of air or gas present in the voids and from the increased solution of gas in the pore water under test pressure. The strength envelopes for partially saturated soil have the curved portion in the low stress range. For partially saturated soils, therefore, this curved portion of the envelope should be used including the cohesion intercept when the dam stresses are in this low range. Because the envelopes are the curved lines, the strength relationship cannot be expressed conveniently in terms of 'cohesion' and 'angle of internal friction' and hence approximation for this condition is necessary. For the purpose of design, therefore, these curved envelopes shall be replaced with a series of straight lines approximately parallel to the curved envelopes so that the cohesion intercept and friction angle can be determined for various normal load ranges.

4.3 R Test

4.3.1 In this test the fully saturated soil sample is allowed to consolidate under one set of normal stress condition and after allowing a proper degree of consolidation it is subjected to shearing process during which the water content is kept constant, that is, no drainage is allowed. Consolidation of sample prior to shearing dissipates the pore pressure due to load and only the pore pressures due to shearing are present at the time of failure. The results of this test, after deducting pore pressure in case of triaxial shear test and without such deduction in case of direct box shear, can be considered as effective stress parameters.

4.3.2 Results of these tests are valid only if full saturation of the soil sample is achieved. For achieving complete saturation the usual method of back pressure is considered satisfactory.

4.4 S Test — This test is performed by consolidating initially the soil sample to the appropriate consolidation stresses and then shearing it very slowly. The slow rate of shearing affords enough time for the pore water to drain out of the voids in the soil mass under shearing stress. Therefore during the process of shearing pore pressures do not get built up. This means that the effective normal stress on the soil sample during shear is equal to the applied normal consolidation stress. In this test the value of the parameter ϕ (angle of internal friction) is greater than that obtained from the same soil for R test, because the neutral stress is practically zero and intergranular friction between the soil particles acts in its full magnitude.

4.5 Choice of Type of Test

4.5.1 Q test approximates the behaviour of earth dams consisting of impervious soils during and immediately after construction. It gives relationship between shear strength and normal pressures in terms of total stresses. This test is also applicable to impervious foundation layer in which the consolidation rate is slow as compared to the fill placement rate. In cases where the foundation soil exists in unsaturated state but is likely to get saturated during construction, due to creation of partial pool or due to any other reason, it is desirable to saturate the samples prior to triaxial shear testing so as to simulate the site conditions and to obtain realistic results. Similar testing is desired if the part of earth dam gets saturated due to creation of partial pool during construction. These test results shall be applicable to foundation and the portion of earth dam likely to get saturated before application of further superimposed load due to raising of earth dam. For other portion the testing shall be done for dam material samples at compaction moisture content and for foundation samples at natural moisture contents.

4.5.2 R test approximates the behaviour during the sudden drawdown of impervious zones and of impervious foundation layers that have consolidated fully by the time the reservoir comes into operation. The test results are also used in analysing the upstream slope during a partial pool condition and downstream slope during steady seepage.

4.5.3 The results of S test are suitable for freely draining soils in which pore pressures do not develop.

4.6 Choice of Test Equipment

4.6.1 As far as possible triaxial tests shall be carried out for foundation and embankment materials.

4.6.2 Direct shear test can be used in respect of S test for freely draining material.

4.7 Use of Ultimate and Peak Shear Strength Values — Normally the peak shear strength values shall be used, however, where the ultimate shear strength shows a drop of more than 20 percent over the peak shear strength, the ultimate shear strength values shall be used.

5. DESIGN CONDITIONS OF ANALYSIS AND ALLOWANCE FOR PORE PRESSURE

5.1 An earth dam shall be safe and stable during all phases of construction and operation of the reservoir. Hence the analysis shall be done for the most critical combination of external forces which are likely to occur in practice. The following conditions are usually critical for the stability of an earth dam:

a) *Case I* — Construction condition with or without partial pool (for upstream and downstream slopes),

b) *Case II* — Reservoir partial pool (for upstream slope),

c) *Case III* — Sudden drawdown (for upstream slope),

d) *Case IV* — Steady seepage (for downstream slope),

e) *Case V* — Steady seepage with sustained rainfall (for downstream slope) (where annual rainfall is 200 cm or more), and

f) *Case VI* — Earthquake condition (for upstream and downstream slopes).

5.1.1 The various design conditions of analysis for upstream and downstream slope along with the minimum values of factors of safety to be aimed at and use of type of shear strength for each condition of analysis are given in Appendix A.

5.2 Case I — Construction Condition With or Without Partial Pool (for Upstream and Downstream Slopes)

5.2.1 This represents a situation when the structure is just constructed. In this condition the pore pressures developed as a result of dam material compression due to the overlying fill are not dissipated or are only partly dissipated. If the rate of raising of dam is less than about 15 metres per year, this condition may not become critical as the residual pore pressure in the dam and foundation are expected to be negligible except in highly clayey foundation with high water table, for example, marshy areas.

5.2.2 Construction pore pressures may exceed the pore pressures likely to be developed due to the seepage from the reservoir and consequently may control the design of dam. The magnitude and distribution of these pore pressures depend primarily on the construction water content in embankment and natural water content in the foundation, the properties of the soil, rate of raising, the height of dam and the rate at which dissipation may occur due to internal drainage.

5.2.3 While examining the stability by effective stress method, the pore pressure shall be accounted for by Hilf's method as given in Appendix B. For the portion of the assumed failure surface passing through the saturated foundation or dam material, total stress analysis using results of undrained triaxial tests may be used. Direct shear test may also be used where other tests are not available. For casing material with large particle size a bigger box may be used. The effect of consolidation, if any, during construction on the shear strength may be neglected.

5.2.4 Moist weights of zones shall be considered in working out resisting and driving forces when the material included in the sliding mass is not likely to get saturated during construction due to the creation of partial pool or due to any other reasons. In other cases where saturation is expected, saturated weights shall be considered while working out driving forces. Contribution of frictional force to the resisting force shall be taken

as nil in case of saturated and unconsolidated material, if total stress analysis is used.

5.2.5 Where conditions of initial filling of reservoir, without dissipation of pore pressures, develop the analysis as given for partial pool in **5.3** shall apply.

5.3 Case II — Reservoir Partial Pool (for Upstream Slope)

5.3.1 This condition corresponds to the initial partial pool filling in which it is assumed that a condition of steady seepage has developed at the intermediate stages. The stability of upstream slope shall be investigated for various reservoir levels on upstream, usually levels corresponding to one-third to two-thirds height of head of water to be stored at full reservoir level and minimum value of factor of safety worked out. The analysis should account for reduction, if any, in the effective normal stresses where the pore pressures developed during construction are not dissipated before a partial pool condition can develop.

5.3.2 All the zones above phreatic line drawn for upstream water level under consideration should be considered as moist for working out resisting and driving forces and zones below it should be taken with their submerged weights for working out both resisting and driving forces.

5.3.3 Partial pool condition may not prove to be critical for all earth dams and hence analysis for this condition needs to be carried out only in cases where it is considered necessary. This condition is likely to be critical in cases of high dams where the range of drawdown is small as compared to the height of dam.

5.4 Case III — Sudden Drawdown (for Upstream Slope)

5.4.1 Earth dams may get saturated due to prolonged higher reservoir levels. Sudden drawdown condition corresponds to the subsequent lowering of reservoir level rate faster than pore water can dissipate. This induces unbalanced seepage forces and excess pore water pressures. This condition becomes critical if the materials of the upstream portion of the dam are not freely draining.

5.4.2 Depending upon the value of the coefficient of permeability of the upstream shell material, the pore pressures in the shell material in the drawdown range shall be allowed arbitrarily in the analysis as follows:

a) Full pore pressures shall be considered if the coefficient of permeability is less than 10^{-4} cm/s.

b) No pore pressures shall be considered if the coefficient of permeability is greater than 10^{-2} cm/s.

c) A linear variation from full pore pressures to zero pore pressures shall be considered for the coefficients of permeability lying between 10^{-4} cm/s to 10^{-2} cm/s.

The above values of pore pressures are based on a drawdown rate of 3 m/month.

5.4.2.1 For the core material which is generally impervious full pore pressures shall be allowed for the core zone lying in the drawdown range. If a zone of random material with the properties intermediate between core and the shell material is provided in between upstream shell and core of the dam, the pore pressures for sudden drawdown condition shall be allowed for in the same way as for the core.

5.4.3 The drawdown pore pressure can be determined in accordance with the method given by A. W. Bishop using the formula:

$$U = \gamma_w [h_c + h_r (1 - n) - h] \text{ (\textit{see} Fig. 1)}$$

where

U = drawdown pore pressure at any point within the core;

γ_w = density of water;

h_c = height of core material at the point;

h_r = height of shell material at the point;

n = specific porosity of shell material, that is, volume of water draining out from the shell per unit volume; and

h = drop in the head under steady seepage condition at the point.

5.4.4 The drawdown pore pressures may also be determined by drawing flow nets.

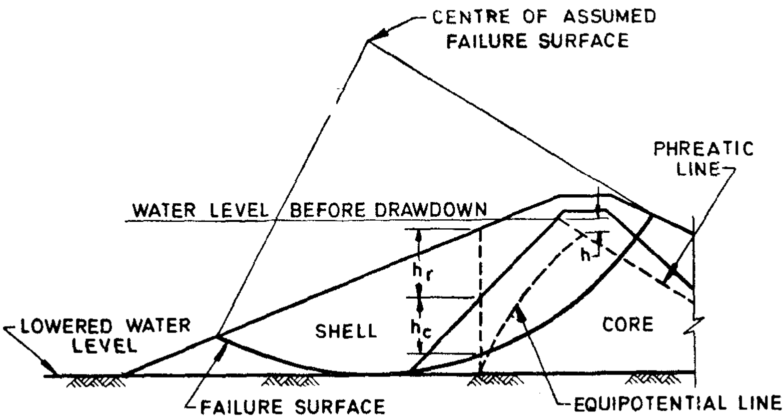


FIG. 1 CRITERION FOR DRAWDOWN PORE PRESSURES IN COMPRESSIBLE CORE

5.4.5 The weights of material to be considered for computing driving and resisting forces shall be as given below:

a) *Zones above phreatic line* — All the zones above phreatic line shall be considered as moist for computing both the driving and resisting forces.

b) *Zones in the drawdown range* — For computing driving forces the core material and non-free-draining material shall be considered as saturated and freely draining material shall be considered as moist. For computing resisting forces the pore pressures shall be adopted as for any of the methods indicated in **5.4.2**, **5.4.3** and **5.4.4**.

c) *Zones below drawdown level* — All zones including foundation zone below the drawdown level shall be considered as buoyant for computing both the driving and resisting forces.

5.4.6 The analysis for upstream slope shall be done for the condition of the drawdown from:

a) Full reservoir level (FRL) in case of earth dams with gated spillways, and

b) A level corresponding to mid-level between full reservoir level (FRL) and maximum water level (MWL) to lowest water level (LWL) in case of earth dams with ungated spillways.

5.4.7 For downstream slope the analysis shall be carried out for the condition of drawdown from maximum tail water level (*Max* TWL) to minimum tail water level (*Min* TWL).

5.5 Case IV — Steady Seepage (for Downstream Slope)

5.5.1 The condition of steady seepage is developed when the water level is maintained at a constant level for sufficiently long time and the seepage lines are established in the earth dam section. This condition is likely to be critical for the downstream slope. In the analysis, existence of tail water and drawdown effects, if any, shall also be taken into account (*see also* **5.4.7**). The stability of downstream slope shall be examined by effective stress method. Steady seepage from level in the reservoir which is sustained for a period of one month should be taken as critical.

5.5.2 The stability analysis of the earth dam shall be done assuming that the dam is fully saturated below phreatic line. Allowance for pore pressure in the analysis shall be made in terms of the buoyancy of the material or by drawing flow nets. The core material lying below the phreatic line (and above the tail water level, if any) shall be considered as saturated for calculating the driving forces and buoyant for resisting forces. All the zones of the dam and foundation lying below the tail water level, if any, shall be considered as buoyant for calculating the driving and resisting

forces. A part of upstream pervious shell material below the phreatic line, if any, included in trial sliding mass shall be considered as saturated for calculating the driving and buoyant for resisting forces. All zones lying above the phreatic line shall be considered as moist for calculating both the driving and resisting forces.

5.5.3 In case of homogeneous dams with no internal drainage the phreatic line will emerge on downstream side in the dam above the ground surface. In such a case the portion of the dam below the phreatic line shall be considered as saturated for calculating driving forces and buoyant for resisting forces.

5.6 Case V — Steady Seepage with Sustained Rainfall (for Downstream Slope)

5.6.1 Where there is a possibility of sustained rainfall, the stability of the downstream slope shall be analysed on arbitrary assumption that a partial saturation of shell material due to rainfall takes place. Accordingly for this condition of analysis, the shell and other material lying above the phreatic line shall be considered as moist for calculating driving forces and buoyant for resisting forces. All the other assumptions regarding allowance for pore pressure shall be in accordance with **5.5.2**.

5.6.2 The saturation for the downstream shell material shall be assumed as under:

a) Fifty percent if the coefficient of permeability is 10^{-4} cm/s or less,

b) Zero percent if the coefficient of permeability is 10^{-2} cm/s or more, and

c) Percentage shall vary linearly from fifty to zero for the coefficients of permeability lying between 10^{-4} cm/s and 10^{-2} cm/s.

5.7 Case VI — Earthquake Condition (for Upstream and Downstream Slopes) — Upstream and downstream slopes shall be analysed for earthquake conditions in accordance with the provisions contained in IS : 1893-1975*.

6. CONSIDERATIONS FOR THE PHREATIC LINE

6.1 Methods for determining the phreatic line are available in any standard book on soil mechanics. It is, however, not necessary to determine accurately the phreatic line for stability analysis. The approximate method given in **6.2** is good enough for the purpose.

6.2 Approximate Method for Determining Phreatic Line — The various assumptions involved in the theory of seepage analysis are seldom

*Criteria for earthquake resistant design of structures (*third revision*).

realized in practice. Hence an approximate method for drawing the phreatic line for the discharge face of the core may be adopted. Instead of drawing a basic prarabola, the phreatic line shall be obtained by joining the entrance point and breakout point on the discharge face. The breakout point is located on discharge face at a height of $h/2$ from base of the dam, h being the head on upstream above the impervious base. A short transition curve may be introduced at the entrance.

7. METHODS OF ANALYSIS

7.1 The methods of analysing the slope stability depending upon the profile of failure surface are:

a) Circular arc method, and

b) Sliding wedge method.

7.2 Circular Arc Method

7.2.1 In this method of analysis the surface of rupture is assumed as cylindrical or in the cross section by an arc of a circle. This method, also known as Swedish or Slip Circle method, is generally applicable for analysing slopes of homogenous earth dams and dams resting on thick deposits of fine grained materials.

7.2.2 For this method, assumptions given at **7.2.2.1** to **7.2.2.4** shall be made.

7.2.2.1 No shearing stresses act across the plane of the cross section and the analysis is treated as two dimensional.

7.2.2.2 The section of the dam analysed is of unit thickness.

7.2.2.3 The sliding mass is divided into a convenient number of slices and each slice is assumed to act independently of its adjoining slices and the forces acting on the sides of a slice have no influence on the shear resistance which may develop on the bottom of the slice.

7.2.2.4 The shear strength of the various zones along the potential failure surface is mobilized simultaneously.

7.2.3 Detailed procedure of analysis shall be as given in Appendix C.

7.3 Sliding Wedge Method

7.3.1 The sliding wedge method of analysis is generally applicable in the circumstances where it appears that the failure surface may be best approximated by series of planes rather than a smooth continuous curve as for the method of circular arc. This method is generally applicable under the following two circumstances:

a) Where one or more horizontal layers of weak soil exist in the upper part of the foundation, and

b) Where the foundation consists of hard stratum through which failure is not anticipated and dam resting on it has a core of fine-grained soil with relatively large shells of dense granular material.

7.3.2 In sliding wedge method of analysis the trial sliding mass is divided into two or three segments, the top segment is called active wedge and the bottom segment is called passive wedge. The middle wedge in case of a three wedge system is called the central block. The resultant of the forces acting on the active wedge and the passive wedge are first determined. These resultants acting on the central block along with other forces on the block shall give a closed polygon for stability. The procedure for this analysis is given in Appendix D.

8. ANALYSIS FOR EARTHQUAKE FORCES

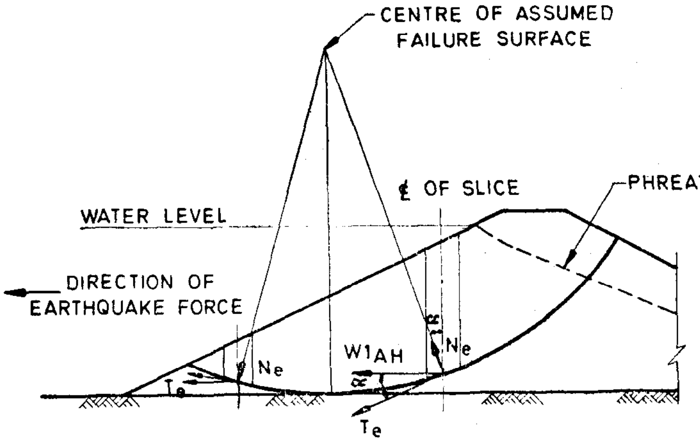
8.1 In regions of seismic activity stability calculations of the slope of a dam should also include seismic forces because they reduce the margin of safety or may even bring about the collapse of the structure. General design approach for earthquake forces is given in IS : 1893-1975*. Where the analysis is carried out by the circular arc or sliding wedge method, total weight of the sliding mass considered for working out horizontal seismic force shall be based on saturated unit weights of the zones below the phreatic line (*see* Fig. 2) and moist weights above it. If the zone above the phreatic line is freely draining, drained weights shall be considered for that zone. Detailed procedure is given in Appendix E for circular arc method and in Appendix F for sliding wedge method.

8.2 As stated in **5.4.1**, the critical condition of analysis of upstream slope for operating condition is the sudden drawdown. When the reservoir is full the seepage pressure acting towards the downstream side increases the resistance of upstream slope towards the sliding and as such, such a condition is not considered critical. However, this condition combined with earthquake forces is considered critical as the stability of upstream slope gets reduced and may lead to the failure of dam when the reservoir is full. Similarly, the downstream slope of the dam shall be analysed for a condition of steady seepage combined with earthquake forces.

9. EVOLVING THE FINAL SECTION

9.1 In determining the stability of the proposed slope it is necessary to try many different circles as one trial arc gives the value of factor of safety for that arc only. The circle which yields the minimum value of factor of safety is the most critical. Theoretically it is necessary to try an infinite number of possible failure circles with different centres and radii before the

*Criteria for earthquake resistant design of structures (*third revision*).



$W1$: Saturated unit weight of the material in the slice (below phreatic line)
A_H : Horizontal earthquake coefficient
T_e : Tangential component of earthquake force (W1.cosα.AH)
N_e : Normal component of earthquake force (W1.sinα.AH)

FIG. 2 ANALYSIS OF EARTHQUAKE FORCES

most critical one giving lowest accepted factor of safety may be located. In practice, however, a limited number of slip circles, about 12 to 15, selected on the basis of past experience are considered sufficient for each condition of analysis.

9.2 After trying about 12 to 15 failure circles, if the lowest value of factor of safety is acceptable the profile of the section needs no changes and the assumed profile shall be considered adequate from considerations of stability. However, if the value of the factor of safety obtained for the critical failure arc is more than required, the section shall be modified by reducing the berm widths and steepening the slopes. The above process shall be repeated till the profile of the section gives the required factor of safety. If, on the other hand, the value of factor of safety obtained during the process of calculation for the failure surface is less than the minimum acceptable, the same shall be increased to the required value by trials after carrying out necessary changes in the profile.

9.3 For the sake of simplicity and reducing the calculations, the various materials, namely, riprap, internal filters, rock-toes, etc, falling within the sliding mass shall be considered to have the same properties as those of the respective zones within which they are located. This will not materially effect the value of factor of safety as these materials usually cover only a small sectional area as compared to the areas of the zones in which

they are located. However, if such materials cover appreciable cross-sectional area, they shall be considered separately.

9.4 Stability analysis of the slopes shall be done for sections of dam for different heights, the entire length being divided into suitable reaches. In deciding the reaches, variations in foundations conditions shall also be taken into account.

10. PRESENTATION OF RESULTS

10.1 All the results of stability analysis for each condition of analysis for the final section evolved shall be presented on one drawing sheet only and shall be called as master sheet. The various circles tried should be numbered serially and shown on this sheet. The critical circle shall be marked distinctly in colour. The design parameters (densities and shear strength values) along with the design assumptions shall be shown on the drawing. A table showing the values of factors of safety for various circles shall also be shown.

10.2 If the topography of the ground across the axis of the dam, on which dam will be resting, is very undulating the profile of the ground along the transverse direction of dam axis shall be plotted and the proposed earth dam section shown on it. The analysis of this section shall then be carried out. If the topography of the ground is fairly uniform and flat, the cross section shall be drawn after considering the horizontal ground profile with its elevation as that of a point along the axis of the dam.

APPENDIX A

(*Clause* 5.1.1)

MINIMUM DESIRED VALUES OF FACTORS OF SAFETY AND TYPE OF SHEAR STRENGTH RECOMMENDED FOR VARIOUS LOADING CONDITIONS

Case No.	*Loading Condition of Dam*	*Slope Most Likely to be Critical*	*Pore Pressure Assumptions*	*Type of Shear Strength Test to be Adopted*	*Minimum Desired Factor of Safety*
I	Construction condition with or without partial pool*	Upstream and downstream	To be accounted for by Hilf's method	Q R†	1·0
II	Reservoir partial pool	Upstream	Weights of material in all zones above phreatic line to be taken as moist and those below as buoyant	R S‡	1·3
III	Sudden drawdown:				
	a) Maximum head water to minimum with tail water at maximum	Upstream	As given in 5.4.2	R S‡	1·3
	b) Maximum tail water to minimum with reservoir full	Downstream	As given in 5.4.5	R S‡	1·3
IV	Steady seepage with reservoir full	Downstream	As given in 5.5.2	R S‡	1·5
V	Steady seepage with sustained rainfall	Downstream	As given in 5.6.1	R S‡	1·3
VI	Earthquake condition:				
	a) Steady seepage	Downstream	As given in case IV	R S‡	1·0§
	b) Reservoir full	Upstream	As given in case II	R S‡	1·0§

NOTE — These factors of safety are applicable for the methods of analysis mentioned in this standard.

*Where the reservoir is likely to be filled immediately after completion of the dam, construction pore pressure would not have dissipated and these should be taken into consideration.

†This is to be adopted for failure plane passing through impervious foundation layer.

‡S test may be adopted only in cases where the material is cohesionless and free draining.

§Values are according to IS : 1893-1975 'Criteria for earthquake resistant design of structures (*third revision*)'.

APPENDIX B

(*Clause* 5.2.3)

COMPUTATION OF PORE PRESSURES BY J. W. HILF'S METHOD

B-1. ASSUMPTIONS

B-1.1 The following assumptions shall be made:

a) Only vertical embankment strain (compression) takes place during construction, there is no lateral bulging;

b) The relationship between embankment compression and effective stress is known;

c) The pressures in the pore water and the air in the pores are always equal and directly after compaction of the embankment material they are equal to atmospheric pressure;

d) The decrease in the embankment volume at any given elevation under the weight of the fill placed above is caused by compression of the air in the voids and solution of the air in the pore water;

e) Boyles's and Henry's laws are valid for this compression and solution; and

f) No dissipation of pore water pressures from drainage occurs during construction.

B-2. FORMULA

B-2.1 Hilf's formula for evaluating pore pressure is

$$U = \frac{P_a \cdot \Delta}{V_a + 0{\cdot}02\ V_w - \Delta}$$

where

U = induced pore pressure in kg/cm^2;

P_a = absolute atmospheric pressure in kg/cm^2;

Δ = embankment compression, in percent of original total embankment volume;

V_a = volume of free air in the voids of the soil directly after compaction, in percent of original total embankment volume; and

V_w = volume of pore water, in percent of original total embankment volume.

B-2.2 The numerical values for the initial air and water volume for use in the formula may be estimated from laboratory compaction tests and from previous experience with average embankment densities. The earth dam compressibility may be derived from laboratory consolidation tests or from the average of field measurements for similar dam materials.

B-2.3 The equation at **B-2.1** may also be used to compute the theoretical pore pressure at which all the air is forced into solution; that is, when the dam becomes completely saturated. This state which occurs when the compression of the specimen Δ becomes equal to the initial volume of air in the voids V_a is described by the equation:

$$U = \frac{P_a . V_a}{0{\cdot}02\ V_w}$$

where the symbols are as described in **B-2.1**.

APPENDIX C

(*Clause* 7.2.3)

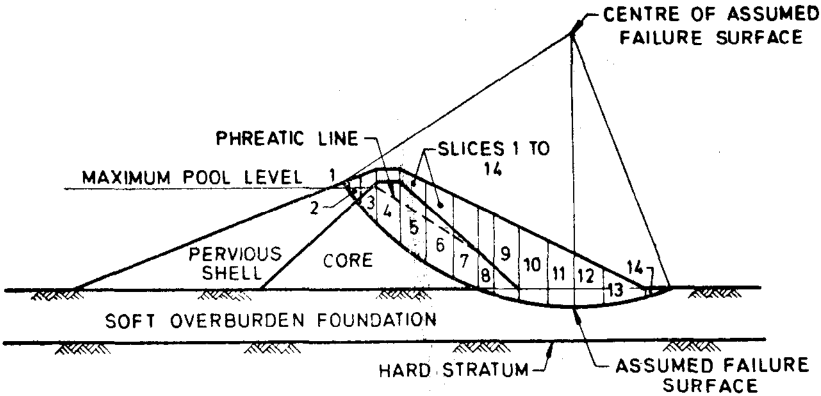
PROCEDURE FOR ANALYSIS OF FORCES BY CIRCULAR ARC METHOD

C-1. GENERAL

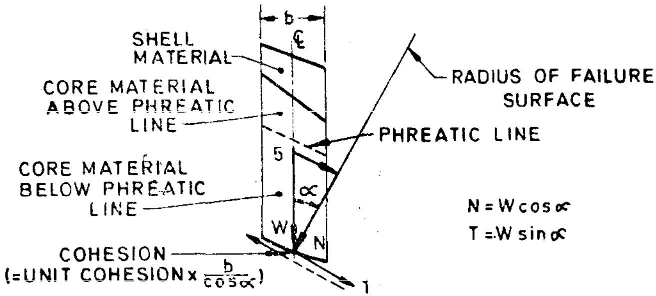
C-1.1 After deciding upon the tentative cross section of the proposed earth dam as stated in 0·6 (d), a possible circular failure surface through the dam and foundation (in case foundation is not firm and through which failure is anticipated) shall be assumed. The trial sliding mass shall be divided into a number of vertical slices. The number of slices depends on the width and profile of the sliding mass, number of various zones included in the sliding mass and the accuracy desired. Usually 10 to 15 slices are desirable. For zoned embankment and stratified foundations with different properties, where an arc of the potential failure surface passes through more than one type of material, the vertical ordinates of the slices for each zone or part of the foundation shall be obtained by locating the slice at each such dividing point. The slices, for convenience, may be of equal width though it is not rigidly necessary to do so. For this, trial surface computations are made of the shear force needed for equilibrium and the strength forces available. The failure surface is shown in Fig. 3A. Forces acting on a typical slice in the analysis of downstream slope for steady seepage condition are shown in Fig. 3B.

C-2. ANALYSIS

C-2.1 The driving forces and resisting forces may be analysed by either of the methods given at **C-2.2** and **C-2.3**.



3A EMBANKMENT SECTION SHOWING SLICES



3B ANALYSIS OF THE FORCES ACTING ON SLICE 5

FIG. 3 CIRCULAR ARC METHOD (METHOD OF SLICES)

C-2.2 Arithmetical Method — The total weight W of the slice of width b is equal to the areas of the various zones included in the slice multiplied by their respective appropriate unit weights (soil plus water). This acts vertically downwards through the centre of gravity of the slice. The two components of this weight W, namely, the force normal to the arc of the slice $N = W \cos \alpha$ and the force tangential to the arc of the slice $T = W \sin \alpha$ are determined after resolving weight W in the radial and tangential direction, α being the angle made by the radius of failure surface with the vertical at the centre of the slice. In the total stress method of analysis the test results include the influence of pore water pressure and hence they need not be accounted for separately in the analysis. However, in the analysis by the effective stress method allowance for pore pressure shall be made separately. The pore water pressure U acting on the arc

of the slice results in an uplift force which reduces the normal component of the weight of the slice. The net or effective downward force acting on the curved bottom boundary of the slice is the total weight minus the upward force due to pore water pressure. The effect of the pore pressure on resisting forces is accounted for by assuming buoyant weight of the material lying below the phreatic line. Component of shearing resistance due to internal friction is therefore $(N - U) \tan \phi$, where ϕ is the angle of shearing resistance of the material at the base of the slice and $(N - U)$ is the effective normal load Ne. Another force acting at the bottom of the slice and which opposes the movement of sliding mass is the shearing resistance offered by the material due to its cohesion, C, and is equal to the unit cohesion, c, multiplied by the length of the bottom of the slice and is approximately equal to $c \times b/\cos \alpha$. In practice the length of the arc may be measured accurately as the expression $b/\cos \alpha$ shall not give the length of arc of the slice when b is not infinitely small. The total resisting or stabilizing force S developed at the bottom of the slice is equal to $(c \times b/\cos \alpha) + (N - U) \tan \phi$. The driving or the actuating force T due to the weight of the slice is equal to $W \sin \alpha$. Similar forces are worked out for all the slices considered for a potential failure surface. The results of these computations shall be tabulated and sums of the resisting and driving forces shall be obtained. The factor of safety against sliding for the assumed failure surface is computed by the equation:

$$FS = \frac{\Sigma S}{\Sigma T} = \frac{\Sigma [\, C + (N - U) \tan \phi \,]}{\Sigma W \sin \alpha}$$

where

FS = factor of safety,

S = resting or stabilizing force,

T = driving or actuating force,

$C = c \times \dfrac{b}{\cos \alpha}$,

N = force normal to the arc of slice,

U = pore water pressure,

ϕ = angle of shearing resistance,

W = weight of the slice,

α = angle made by the radius of the failure surface with the vertical at the centre of slice,

c = unit cohesion, and

b = width of slice.

C-2.3 Graphical Method — The summations of the resisting and driving forces may be done by graphical method and may be adopted in preference to the arithmetical method described at **C-2.2**. Any vertical line within sliding mass from the outer slope of the dam to the bottom of failure surface represents weight of strip or slice infinitely small in width. This is resolved into two components, one normal to the failure surface N and other tangential to it T. These components for other various vertical slices selected within the failure are plotted separately on two horizontal base lines after projecting the verticals of the failure surface on the base lines. The extremities of these normal and tangential components are then joined by smooth and continuous curves. The areas under these curves represent the summation of the normal and tangential forces acting on the failure surface. The areas for various zones are planimetered and multiplied by the respective unit weights of the material. In order to account for the effect of the pore pressure, the normal forces shall be worked out on the basis of effective unit weights. The summation of normal forces when multiplied by the respective tangent of angle of internal friction along with addition of cohesion gives the total resisting force. The summation of tangential forces gives total driving force. The factor of safety shall then be calculated as given in **C-2.2**.

C-2.4 Typical calculations for working out the factor of safety of downstream slope for a condition of steady seepage as given by arithmetical method (*see* **C-2.2**) are given in Tables 1, 2 and 3 and Fig. 4 while those by graphical method (*see* **C-2.3**) are given in Tables 4, 5 and 6 and Fig. 5. The final results of the computations by both the methods are practically the same.

TABLE 1 ADOPTED DESIGN DATA

Zone	R = Strength		Unit Weight, t/m³		
	c, t/m^2	tan ϕ	Moist	Submerged	Saturated
Shell	0	0·55	2·05	—	—
Core	2·0	0·35	1·76	0·905	1·905
Foundation	1·9	0·45	—	0·744	—

TABLE 2 CALCULATIONS FOR AREA OF SLICES

(*Clause* C-2.4)

Slice	Horizontal Width	Shell Above Phreatic Line				Core Above Phreatic Line				Core Below Phreatic Line				Foundation			
		Height of Slice			Area	Height of Slice			Area	Height of Slice			Area	Height of Slice			Area
		Left	Right	Avg		Left	Right	Avg		Left	Right	Avg		Left	Right	Avg	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
	m	m	m	m	m^2	m	m	m	m^2	m	m	m	m^2	m	m	m	m^2
1	1·55	0·0	4·0	2·0	3·10	—	—	—	—	—	—	—	—	—	—	—	—
2	3·75	4·0	2·0	3·0	11·25	0	3·25	1·62	6·07	0	3·50	1·75	6·56	—	—	—	—
3	3·75	2·0	4·0	3·0	11·25	3·25	4·50	3·87	14·51	3·50	5·50	4·50	16·87	—	—	—	—
4	4·50	4·0	6·0	5·0	22·50	4·50	2·75	3·62	16·29	5·50	7·50	6·50	29·25	—	—	—	—
5	4·25	6·0	8·5	7·25	30·81	2·75	1·50	2·12	9·01	7·50	8·50	8·0	34·00	—	—	—	—
6	3·25	8·5	10·0	9·25	30·06	1·50	0·0	0·75	2·43	8·50	6·50	7·50	34·37	0·0	2·0	1·0	3·25
7	6·75	10·0	3·5	11·75	79·31	—	—	—	—	6·50	0	3·25	21·93	2·0	6·0	4·0	27·00
8	6·00	13·5	13·5	13·5	81·00	—	—	—	—	—	—	—	—	6·0	8·0	7·0	42·00
9	9·00	13·5	10·5	12·0	108·00	—	—	—	—	—	—	—	—	8·0	10·25	9·12	82·08
10	8·00	10·5	8·0	9·25	74·00	—	—	—	—	—	—	—	—	10·25	11·0	10·62	84·96
11	7·75	8·0	5·5	6·75	52·31	—	—	—	—	—	—	—	—	11·0	10·5	10·75	83·31
12	7·00	5·5	3·0	4·25	29·75	—	—	—	—	—	—	—	—	10·5	9·0	9·75	68·25
13	7·00	3·0	0·0	1·50	10·5	—	—	—	—	—	—	—	—	9·0	6·5	7·75	54·25
14	11·00	—	—	—	—	—	—	—	—	—	—	—	—	6·5	0·0	3·25	35·75

TABLE 3 COMPUTATIONS FOR RESISTING AND DRIVING FORCES

(*Clause* C-2.4)

	AB	BC				CD									Total
1. Portion	AB	BC				CD									
2. Slice	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
3. Area of Various Zones in the Slice, m² — Shell	3·1	11·25	11·25	22·50	30·81	30·06	79·31	81·00	108·00	74·00	52·31	29·75	10·50	—	
Core — Above phreatic line	—	6·07	14·51	16·29	9·01	2·43	—	—	—	—	—	—	—	—	
Core — Below phreatic line	—	6·56	16·87	29·25	34·00	24·37	21·93	—	—	—	—	—	—	—	
Foundation	—	—	—	—	—	3·25	27·00	42·00	82·08	84·96	83·31	68·25	54·25	35·75	
4. Weight for Resisting Forces, t — Shell	6·35	23·06	23·06	46·12	63·16	61·62	162·58	166·05	221·40	151·70	107·23	60·98	21·52	—	
Core — Above phreatic line	—	10·68	25·53	28·67	15·85	4·27	—	—	—	—	—	—	—	—	
Core — Below phreatic line	—	5·93	15·26	26·47	30·77	22·05	19·84	—	—	—	—	—	—	—	
Foundation	—	—	—	—	—	2·41	20·08	31·24	61·06	63·21	61·98	50·77	40·36	26·59	
Total Weight of Slice (WR)	6·35	39·67	63·85	101·26	109·78	90·35	202·50	197·29	282·46	241·91	169·21	111·75	61·88	26·59	
5. Weight for Driving Forces, t — Shell	6·35	23·06	23·06	46·12	63·16	61·62	162·58	166·05	221·40	151·70	107·23	60·98	21·52	—	
Core — Above phreatic line	—	10·68	25·53	28·67	15·85	4·27	—	—	—	—	—	—	—	—	
Core — Below phreatic line	—	12·40	32·13	55·72	64·77	46·42	41·77	—	—	—	—	—	—	—	
Foundation	—	—	—	—	—	2·41	20·08	31·24	61·05	63·21	61·98	50·77	40·36	26·59	
Total Weight of Slice (WD)	6·35	46·23	80·72	130·51	143·78	174·72	224·43	197·29	282·46	214·91	169·21	111·75	61·88	26·59	
6. α, degree	66	60	53·30	47	40	35	29	22·30	13	4·30	4·30	12·30	20	31·00	
7. cos α	0·406	0·500	0·594	0·682	0·766	0·819	0·874	0·923	0·974	0·996	0·996	0·976	0·939	0·857	
8. sin α	0·913	0·866	0·803	0·731	0·642	0·573	0·484	0·382	0·225	0·078	0·078	0·216	0·342	0·515	
9. Effective Normal Force (Ne) = WR. cos α,t	2·57	19·83	37·92	69·05	84·09	73·99	176·98	182·09	275·11	214·05	168·53	109·06	58·10	22·78	
10. Driving Force (T) = WD. sin α, t	5·79	39·59	64·81	95·40	92·30	65·73	108·62	75·36	63·55	16·76	−13·19	−24·13	−21·16	−13·69	
11. Σ Ne	2·57			210·89						1280·69					
12. Σ T	5·79			292·10						257·85					555·74
13. tan φ	0·55			0·35						0·45					
14. Σ Ne. tan φ, t	1·41			73·81						576·31					651·53
15. Cohesion, C — Unit Cohesion, c	0			2·0						1·9					
Length of Arc, L	4			25						71					
c.L	0			50·0						134·90					184·90

Total Resisting Force = 651·53 + 184·90 = 836·43 t

Factor of Safety $= \frac{836{\cdot}43}{555{\cdot}74} = 1{\cdot}505$

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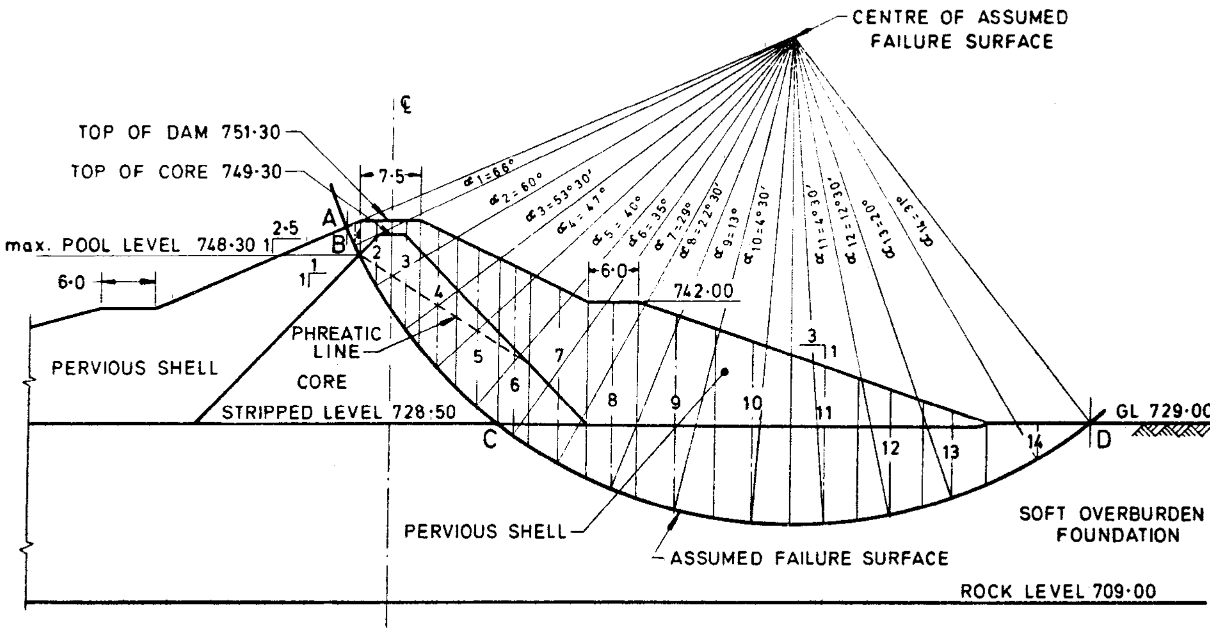


CIRCULAR ARC METHOD

All dimensions in metres.

FIG. 4 TYPICAL CALCULATIONS FOR DOWNSTREAM SLOPE BY ANALYTICAL METHOD (STEADY SEEPAGE)

TABLE 4 ADOPTED DESIGN DATA

(*Clause* C-2.4)

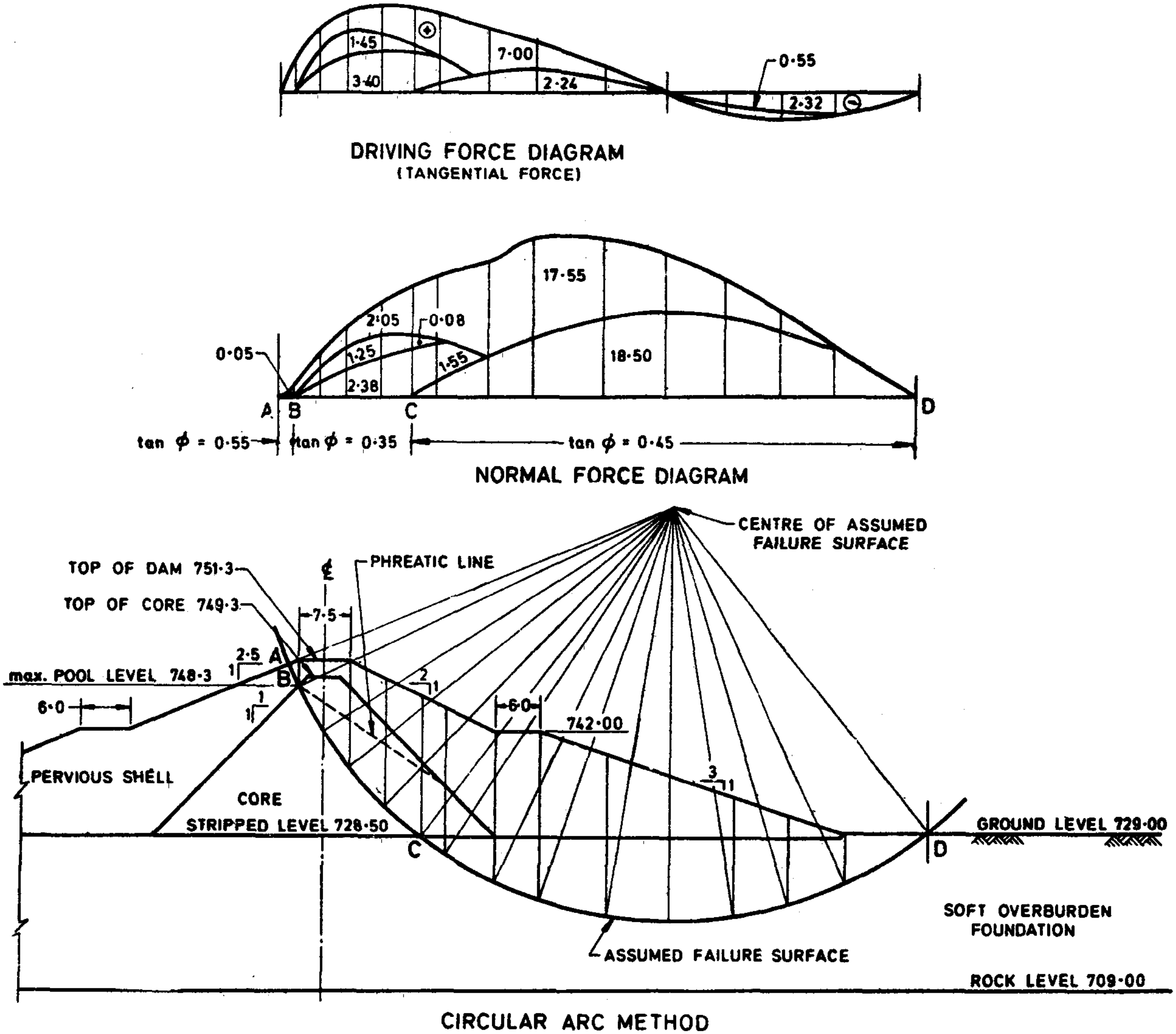
Zone	R Strength		Unit Weight, t/m³		
	c, t/m²	tan φ	Moist	Submerged	Saturated
Shell	0	0·55	2·05 (Dry)	—	—
Core	2·0	0·35	1·76	0·905	1·905
Foundation	1·9	0·45	—	0·744	—

TABLE 5 COMPUTATIONS FOR RESISTING FORCES

(*Clause* C-2.4)

Portion	Zone	Area of Zone		Unit Weight	Normal Force (Effective)	Total Normal Force (Ne) (Effective)	tan φ	Ne. tan φ	Cohesion, C		
		cm²	m²						Unit Cohesion (c)	Length of Arc (L)	Cohesion c.L, t
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
AB	Shell above phreatic line	0·05	1·25	2·05	2·56	2·56	0·55	1·40	0	4	0·00
BC	Core below phreatic line	2·38	59·50	0·905	53·84						
	Core above phreatic line	1·25	31·25	1·76	55·00						
	Shell above phreatic line	2·05	51·25	2·05	105·06	213·90	0·35	74·86	2·0	25	50·00
CD	Foundation	18·50	462·50	0·744	344·10						
	Core below phreatic line	1·55	38·75	0·905	35·07						
	Core above phreatic line	0·08	2·00	1·76	3·52						
	Shell above phreatic line	17·55	438·75	2·05	899·43	1282·12	0·45	576·95	1·9	71	134·90
							Total	653·21			184·90

Total Resisting Force = 653·21 + 184·90
= 838·11 t



All dimensions in metres.

FIG. 5 TYPICAL CALCULATIONS FOR DOWNSTREAM SLOPE BY GRAPHICAL METHOD (STEADY SEEPAGE)

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TABLE 6 COMPUTATIONS FOR DRIVING FORCES

(*Clause* C-2.4)

Zone	Area of Zone		Unit Weight	Driving Force
	cm^2	m^2	t/m^3	t
(1)	(2)	(3)	(4)	(5)
Shell above phreatic line (7·00-0·55)	6·45	161·25	2·05	330·56
Core above phreatic line	1·45	36·25	1·76	63·80
Core below phreatic line	3·40	85·00	1·905	161·92
Foundation (2·24-2·32)	—0·08	—2·00	0·744	—1·48

Total = 554·80t

$$\text{Factor of Safety} = \frac{838{\cdot}11}{554{\cdot}80} = 1{\cdot}51$$

APPENDIX D

(*Clause* 7.3.2)

PROCEDURE FOR ANALYSIS OF FORCES BY SLIDING WEDGE METHOD

D-1. NOTATIONS

D-1.1 For the purpose of this appendix the following notations shall have the meaning indicated after each:

C = total cohesive force,

C_A = developed cohesive force of active wedge,

C_{CB} = developed cohesive force of central block,

CD = developed cohesive force,

C_P = developed cohesive force of passive wedge,

E_A = resultant force of active wedge,

E_P = resultant force of passive wedge,

ΔEH = force required to close the force polygon,

F_A = resultant of normal and frictional force of active wedge,

F_{CB} = resultant of normal and frictional force of central block,

F_P = resultant of normal and frictional force of passive wedge,

ND = developed normal force,

SD = developed shear strength = $CD + (ND - U) \tan \phi D$,

U = pore water pressure,

W_A = total weight of active wedge,

W_{CB} = total weight of central block,
W_P = total weight of passive wedge,
ϕ = angle of internal friction, and
ϕD = developed angle of internal friction required for equilibrium.

D-2. ANALYSIS

D-2.1 An arbitrary failure surface *A, B, C, D* in Fig. 6 is chosen for analysis. Vertical boundaries are assumed between the central block and active and passive wedge.

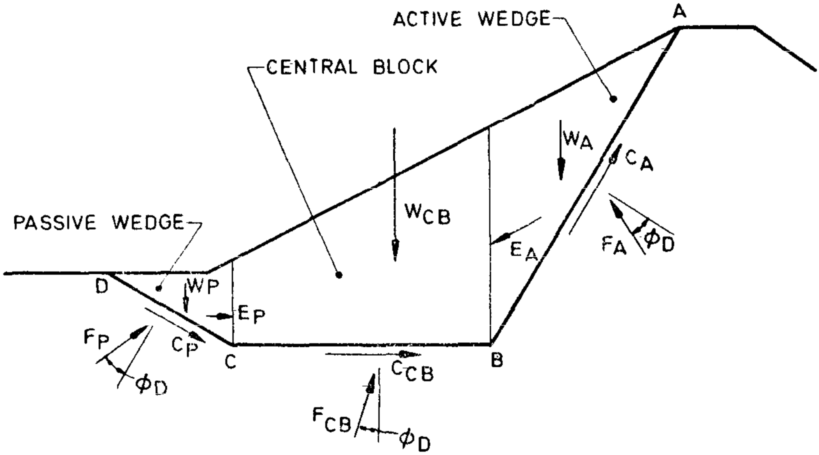

FIG. 6 EMBANKMENT SECTION SHOWING WEDGES

D-2.2 The total weight *W* of the segment is equal to the area multiplied by the appropriate gross unit weight of the material. It is assumed that the shear strength developed at the bottom of wedge is equal to $SD = CD + (ND - U)\tan\phi D$ where $CD = \frac{C}{FS}$, $\tan\phi D = \frac{\tan\phi}{FS}$, and *FS* is a trial factor of safety. *ND* is the normal force acting on the bottom when the wedge is at a state of equilibrium and not at failure. The forces on each segment are considered separately. The developed values of cohesion, *CD*, and developed angle of internal friction, ϕD, along the failure surface are controlled by the assumed trial factor of safety. The magnitude of resultant earth forces E_A and E_P also depend on the value of trial factor of safety. However, the direction of forces acting on the vertical boundaries must be assumed. If the vertical boundary between the active wedge and the central block or passive wedge is located at or below the centre of embankment slope, the direction of the force E_A is assumed to be

parallel to the slope. If the vertical boundary is located near the top of the embankment slope horizontal direction for E_A should be assumed. For a vertical boundary between the centre and top of the slope, an intermediate angle may be selected. The direction of the force E_P in the vertical boundary between the passive wedge and the central block should be assumed parallel to the outer embankment slope if the boundary is located near the centre of the slope. If it is located at or near the toe of the slope the direction of E_P should be assumed as horizontal. The resultant forces acting at the vertical boundaries of the active and passive wedges are determined by constructing force polygon as shown in Fig. 7 and 8. These forces are finally incorporated in the force polygon for the central block (*see* Fig. 9).

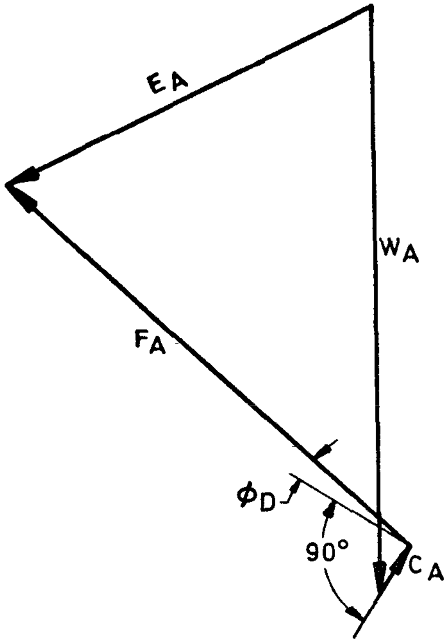

FIG. 7 FORCE POLYGON FOR ACTIVE WEDGE

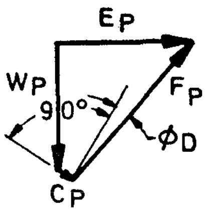

FIG. 8 FORCE POLYGON FOR PASSIVE WEDGE

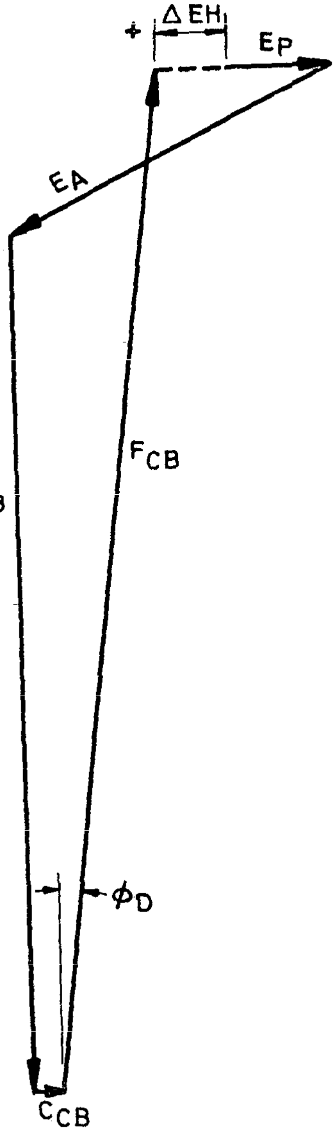

FIG. 9 FORCE POLYGON FOR CENTRAL BLOCK

D-2.3 A condition of equilibrium will generally not be obtained on first trial and several trial analysis with different factors necessary to close the polygon (*see* Fig. 9) is denoted by ΔEH.

D-2.4 The force ΔEH is assumed to act horizontally and its magnitude and sign vary with the trial value of factor of safety. A plot is made of ΔEH *versus* the trial values of factor of safety as shown in Fig. 10 to determine the value of factor of safety for which ΔEH is zero. This factor of safety is required to balance the forces for the sliding surface being analysed. Various trial locations of the active and passive wedges are required to determine the minimum value of factor of safety.

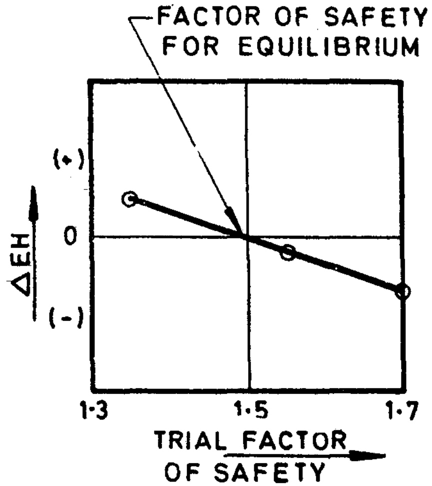

FIG. 10 TRIAL FACTOR OF SAFETY *versus* ΔEH

APPENDIX E

(*Clause* 8.1)

ANALYSIS FOR EARTHQUAKE CONDITION BY CIRCULAR ARC METHOD

E-1. GENERAL

E-1.1 Earthquake forces acting on typical slices of a homogenous dam, one located on the right side and other located on left of the centre of circle of upstream failure surface are shown in Fig. 2. The tangential components of these earthquake forces for all the slices act in the direction of the slide and thus they aggravate the failure. This force shall be added to the

driving forces as worked out in Appendix C, for reservoir operating conditions to get total driving force for earthquake condition. Similarly the overall effect of the normal components of the earthquake forces over a sliding mass is to reduce the normal force worked out for the reservoir operating condition (without earthquake).

E-2. ANALYSIS

E-2.1 The factor of safety for earthquake condition shall be worked out from the following formula:

$$FS = \frac{\Sigma\,[\,C + (\,N - U\,)\tan\phi\,] - \Sigma\,(\,W1 \sin\alpha \tan\phi \times AH\,)}{\Sigma\, W \sin\alpha + \Sigma\, W1 \cos\alpha .\, AH}$$

where

FS = factor of safety;

C = cohesive resistance of the slice;

N = force normal to the arc of slice;

U = pore water pressure;

$(N\text{-}U)$ = effective normal force acting on the failure surface of slice;

ϕ = angle of internal friction;

$W1$ = saturated weight of the slice if it is below phreatic line and moist weights (or drained weights, if it is freely draining) if it is above it;

α = angle between the centre of the slice and radius of failure surface;

AH = horizontal seismic coefficient; and

W = weight of the slice considered for driving force.

APPENDIX F

(*Clause* 8.1)

ANALYSIS FOR EARTHQUAKE CONDITION BY SLIDING WEDGE METHOD

F-1. ASSUMPTIONS

F-1.1 For illustrating a typical example of calculations earthquake forces have been assumed to be of magnitude 0·1 *g*, that is, 10 percent of the weight and acting in the direction of slide.

F-2. NOTATIONS

F-2.1 In addition to the notations given in Appendix D, the following notations shall have the meaning shown against them:

Fh_A = horizontal earthquake force due to weight of active wedge,

Fh_{CB} = horizontal earthquake force due to weight of central block, and

Fh_P = horizontal earthquake force due to weight of passive wedge.

F-3. ANALYSIS

F-3.1 The slope shall be analysed in accordance with the procedure prescribed in Appendix D incorporating the earthquake forces at relevant places. Typical example is illustrated in Fig. 11 to 15.

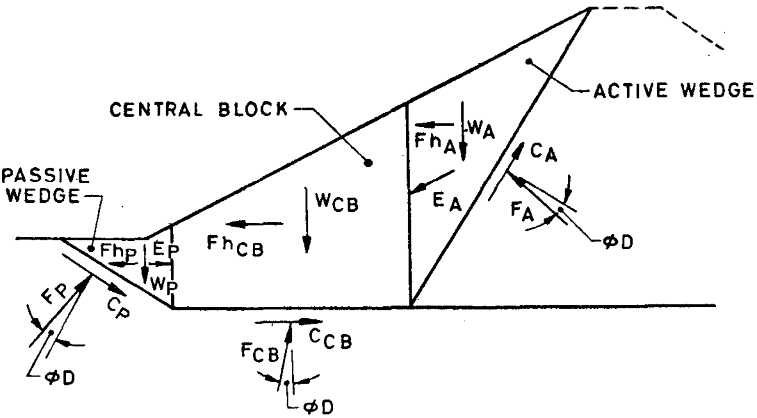

FIG. 11 DAM SECTION SHOWING WEDGES AND EARTHQUAKE FORCES

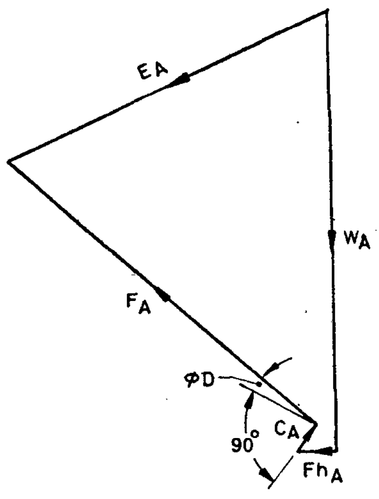

FIG. 12 FORCE POLYGON-ACTIVE WEDGE

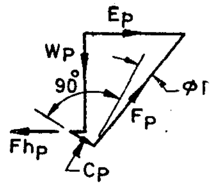

FIG. 13 FORCE POLYGON-PASSIVE WEDGE

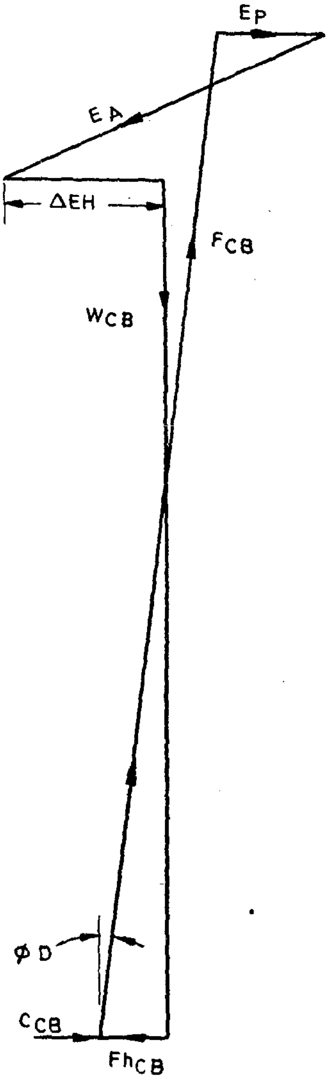

FIG. 14 FORCE POLYGON CENTRAL BLOCK

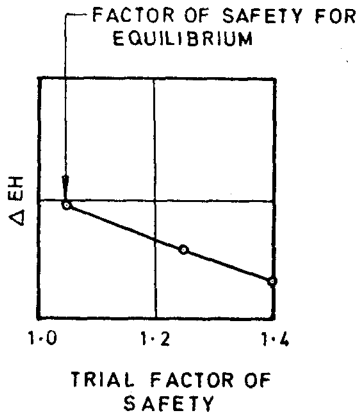

Fig. 15 Trial Factor of Safety *versus* ΔEH

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